Seismic Criteria For California Marine Oil Terminals

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Preface

This document presents guidance on the seismic design of marine oil terminals. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected. The goals are to:

Ensure safe and pollution-free transfer of petroleum products between the ship and land based facilities.
Ensure the best achievable protection of the public health, safety and the environment
Maximize the utilization of limited resources

This document develops and expands on work that was begun by the US Navy to provide seismic design criteria for waterfront construction. It presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals. As such it recognizes the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State and the economics of operating a commercial facility in a competitive environment. Readers must recognize that this standard can not guarantee that if implemented and followed that all damaging effects will be precluded. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The establishment of design levels is more of a management decision that an engineering one. Considering the size of the State of California and the health and economic needs of its inhabitants, this guide is thought to be set at an optimal balance. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage. It consists of a criteria section, supporting technical commentary and three appendices.

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INTRODUCTION

The California State Lands Commission (SLC) was created by the California Legislature in 1938 as an independent body, composed of three members-the Lieutenant Governor and State Controller, both statewide elected officials, and the Director of Finance, an appointee of the Governor. The SLC was given the authority and responsibility to manage and protect the important natural and cultural resources on public lands within the state and the public's rights to access to such lands. In managing the state's lands, the SLC provides two functions: (1) generating revenue for the state, and (2) protecting, preserving and restoring the natural values of state lands. The resources managed by the SLC are diverse and range from commercially valuable minerals such as oil, natural gas, hard rock minerals, sand, gravel, and geothermal steam to unique natural resources such as forests, grazing lands, wetlands, riparian vegetation, and fish and wildlife habitat.

The SLC's Marine Facilities Inspection and Management Division was created in response to the passage of the Lempert-Keen-Seastrand Oil Spill Prevention and Response Act of 1990 which mandated the best achievable protection of California's marine environment and created a $500 million Oil Spill Contingency Fund to help finance emergency response efforts and provide disaster relief in the event of a major oil spill. This strong mandate reflected the Legislature's recognition of the public outcry for stronger environmental protection following the tragedies caused by the MIT Exxon Valdez grounding in Alaska and the MIT American Trader oil spill off Huntington Beach. The SLC is focussed on protecting the marine environment through the prevention of oil spills because no matter how quickly the response is to an oil spill, severe and often irreparable damage occurs to the marine environment. Prevention is the least expensive form of environmental protection. Comprehensive Marine Terminal Regulations were formulated by the SLC and implemented in late 1992. The implementation of these regulations and the SLC inspection activities were responsible in reducing regulatory deficiencies. This document presents guidance on the seismic design of marine oil terminals. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected. The goals are to:

Ensure safe and pollution-free transfer of petroleum products between the ship and land based facilities.
Ensure the best achievable protection of the public health, safety and the environment
Maximize the utilization of limited resources

A typical Marine Oil Terminal (MOT) includes some or all of the following components:

Pier
Wharf and dike
Bulkheads, quay walls, sheet piling
Pipeline (to the first valve inside an EPA containment area)
Pipeline supports
Bumper fendering, camels, batter piles
Mooring components including breasting and mooring dolphins and onshore dead-men
Local on pier in terminal storage tanks (but not storage tanks in backland ashore)
Hose Fuel Transfer equipment and structures
Vapor control systems
Fire suppression and detection systems
Building and other structures on the pier or wharf
Ancillary components
Riprap

Safe, effective seismic design consists of three elements - establishment of performance goals, specification of the earthquake intensity, and definition of the acceptable structural response limits corresponding to the performance goals. Although seismic load-performance requirements exist for structures and bridges, no requirements have been developed for the waterfront structures that are common at ports and marine oil terminals. There is also a lack of geotechnical guidelines for the seismic evaluation and design of waterfront structures. For example, very few standards exist for defining acceptable factors of safety against liquefaction in soil, a major cause of damage at the waterfront. While structural and geotechnical analysis tools for evaluating the occurrence of liquefaction and the response of structures currently exists, guidance standards which define what constitutes acceptable behavior under a prescribed load level have not been established.

It is important to understand that a complete design standard is composed of three major parts:

1. Development of a set of performance goals defining levels of operation required after earthquake ground motions of varying intensity and duration.

2. Specification of a set of earthquake intensities corresponding to prescribed risk levels.

3. Determination of the structural response limits at the specified seismic intensities which will ensure that damage is limited to meet the expected performance levels.

Thus, full design criteria includes definition of :

1. Performance objectives,
2. Specification of ground motion,
3. Specification of analysis procedures,
4. Evaluation of all possible failure modes of the global structure, including the soil foundation,
5. Definition of component damage mechanisms for all elements of the structure,
6. Development of allowable response limits such as strains, ductilities, and drifts to control element damage and structure performance,
7. Evaluation of economics of design,
8. Understanding of the reliability associated with definition of seismic intensity and structural performance.

In general, a reliability analysis evaluates the loading conditions with their measure of uncertainty, and the composition of the structure in terms of material properties, structural member sections used, the uncertainties in materials and construction etc. From the quantification of uncertainty one can calculate the distribution of possible performance outcomes. At the current state of knowledge, full explicit reliability analysis is an unrealistic goal. Generally implicit consideration of reliability aspects is made by coupling a high estimate of expected ground motion with a conservative estimate of structural limit states to ensure that the probability of exceedance of the limit state under the design intensity is sufficiently low.

This document develops and expands on work that was begun by the US Navy to provide seismic design criteria for waterfront construction. This report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals. As such it recognizes the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State and the economics of operating a commercial facility in a competitive structure. Readers must recognize that this standard can not guarantee that if implemented and followed that all damaging effects will be precluded. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The establishment of design levels is more of a management decision that an engineering one. Considering the size of the State of California and the health and economic needs of its inhabitants, this guide is thought to be set at an optimal balance. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage.

DEFINITIONS AND GENERAL CRITERIA

Construction Categories

Ordinary- General normal construction where operational, special enhanced life safety provisions, and spill containment factors are not involved.

Waterfront Transfer Structures- Piers and wharves directly involved in hazardous material transfer.

Essential- Facilities and component elements directly controlling operations that are required for safe operation and plant shutdown. Such facilities must operate during and after an earthquake to the extent required to control operation.

Hazardous Material Containment- Facilities and components serving to prevent the uncontrolled release of hazardous materials. These systems may be composed of a single system or a duel system with secondary containment.

Design Earthquake Levels

This report will utilize the following earthquake levels as defined events.

Level 1- An earthquake with a 50 percent probability of exceedance in 50 years exposure. This event has a return time of 72 years and is considered a **moderate** event likely to occur one or more times during the life of the facility. Such an event is considered a strength event.

Level 2- An earthquake with a 10 percent probability of exceedance in 50 years exposure. This event has a return time of 475 years and is considered a **major** event. Such an event is considered a strength and ductility event.

Level 3- An earthquake with a 5 percent probability of exceedance in 50 years exposure. This event has a return time of 949 years and is considered a **rare** event. Such an event is considered a strength and ductility event.

Level 4- An earthquake with a 3 percent probability of exceedance in 50 years exposure. This event has a return time of 1641 years and is considered a **very rare** event. Such an event is considered a containment event. **Note where ground motions from a 1641-year event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows actual return times and nominal return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	50	72	100
10	50	475	500
5	50	975	1000
3	50	1641	1700

Spill Size

Minor Spill- A spill of less than 1200 barrels of petroleum products or comparable hazardous material.

Major Spill- A spill of 1200 barrels or more of petroleum products or hazardous material.

General Performance Goals

Marine oil terminal facilities designed under this criteria are expected to perform in the following manner:

To resist earthquakes of moderate size which can be expected to occur one or more times during the life of the structure without structural damage of significance. The facility is not expected to sustain a major interruption in operations.

To resist major earthquakes which are considered as infrequent events maintaining life safety, precluding total collapse but allowing a measure of controlled inelastic behavior which will require repair.

To have essential facility components required for safe operation, shutdown, and emergency operations function during and after rare earthquakes

To preclude major spills of hazardous and polluting materials during and after very rare earthquakes.

To utilize economic/risk analysis as an aid in decision making including determining the condition of the facility and it's remaining useful life.

To consider the facility as a system and include the effect of all hazards on the operation of the whole facility and all subcomponents including lifelines.

Inherent in the general performance criteria are the three issues of structural response and integrity, spill containment, and functionality of essential emergency components. The first issue of structural response and integrity refers to the requirement for key elements such as piers and wharves that these structure should not only not collapse but that they must be able to perform to a deformation response limit so as to be in a condition which is repairable. The issue of containment refers to the need to preclude large spills. This can be accomplished by providing segmentation valves and secondary containment devices to limit the maximum size of the spill and containment or by strengthening primary components. The last issue of functionality controls the design of emergency components which are needed for post-earthquake control of the facility.

It should be noted that conformance to this criteria does not guarantee that significant damage will not occur. It does provide a prudent allocation of resources using the best available knowledge at the time it was written. A criterion must have sufficient prescription to serve as a minimum requirement and yet sufficient flexibility to allow for project specific considerations on issues such as the remaining useful life of an existing structure and the allocation of resources in achieving mandated requirements.

It is important that all interested parties including the State, the facility operator and concerned citizens establish a consensus in selecting design levels. The operator must recognize that safe design is in his long term interest by insuring minimization of damage and downtime. The State must recognize its requirement in providing clear minimum acceptable standards which are achievable. Concerned citizens must recognize that resources are sometimes limited and that transfer of oil is vital to the day-to-day life of the State and its economic viability. This document is presented as the first step in achieving that balance.

GROUND MOTION CRITERIA

A probabilistic site seismicity study is required for determining the ground motion associated with analysis of marine oil terminals. The objective of a seismicity study is to quantify the characteristics of ground shaking and the recurrence of potentially damaging ground motions that pose a risk at the site of interest. The approach taken in engineering practice is to use the historical epicenter data base in conjunction with available geologic data to form a best estimate of the probability distribution of site ground motion. Acceptable procedures for conducting a site seismicity study must include the following elements. The process consists of building a model of the region to capture the seismic activity using probabilistic procedures. The procedure consists of:

- Evaluating the regional tectonics and geologic settings
- Determining and defining seismic sources in the region of interest
- Estimating the seismic slip rate along faults in the region
- Defining the study boundaries beyond which earthquakes pose no significant damage potential to the site
- Developing an epicenter data base of historical earthquakes in the region

of interest

- Specifying and formulating the site seismicity/faulting model
- Developing the earthquake (regional and fault specific) recurrence models
- Determining the maximum credible earthquakes for specific source
- Selecting appropriate ground motion attenuation relationship
- Computing the contribution of individual faults or source zones to ground motion estimates
- Combining the source contributions for all faults
- Developing probability distribution for firm site
- Determining local site soil conditions
- Determining the local site response
- Developing site specific time histories or response spectra for causative events

The supporting technical material found in Chapter 1 will present a summary and discussion of the technology for each of the elements of the analysis. Recognition of previous research in establishing recurrence parameters shall be used where available. Such bodies of knowledge are available for California from the California Division of Mines and Geology (CDMG) Internet site. Geologic slip rate data is available for a number of western faults.

Local Site Amplification As a minimum, a one-dimensional equivalent linear or fully nonlinear dynamic soil analysis shall be used to evaluate local site amplification and to determine the modification of the rock spectrum by local soil deposits. A shear-beam model representing the ground conditions from bedrock to surface is typically used, with input of the acceleration time history corresponding to the bottom boundary of the model. When the bedrock boundary slopes steeply in the vicinity of the site, such one-dimensional techniques may be inadequate

Study Results The results of a seismicity study shall include the probability of site ground motion adjusted for local site effects. The results should include a set of earthquakes including magnitude, location, and site acceleration to serve as a set of scenario events in evaluation of damage potential The structural design engineer may use either response spectra or time history techniques in the analysis of a structure.

STRUCTURAL CRITERIA FOR PIERS AND WHARVES

Performance Goals

The criteria are intended to produce a level of design in piers and wharves such that there is a high probability the structures will perform at satisfactory levels throughout their design life.

To resist earthquakes of moderate size which can be expected to occur one or more times during the life of the structure without structural damage of significance.

To resist major earthquakes which are considered as infrequent rare events maintaining life safety, precluding total collapse but allowing a measure of controlled inelastic behavior which will require repair.

To preclude major spills of hazardous and polluting materials for very rare earthquakes.

To utilize economic/risk analysis to consider alternative design

To consider geologic hazards (e.g., liquefaction, slope stability, excessive ground settlement) as a major waterfront problem. The designer shall consider potential ground failures in the design of the structures and account for geotechnical earthquake engineering issues (change in lateral earth pressures, potential lateral movements and increased settlements).

Design Earthquakes

The pier or wharf structure shall be designed to resist the loading produced by:

A Level 1 earthquake

A Level 2 earthquake

In addition containment to preclude a major spill shall be provided for:

A Level 4 earthquake

All crane rails shall be supported on piles including the seaward and the landward rail. The crane rails shall be connected horizontally by a continuous deck, beam or other means to control the gage of the rails and prevent spreading. The rails shall be grounded. For corrosion protection, it is advantageous to insulate the reinforcing steel in the piles from that in the deck.

Piers and wharves containing fueling systems shall be evaluated for a Level 4 earthquake to insure that a major spill of hazardous material is precluded. This may be accomplished by providing secondary containment systems or shutoff valves should there be breaks in fuel lines or primary containment system elements or by strengthening these elements.

Preclude release of hazardous and polluting materials causing a major spill for a Level 4 event

Design Performance Limit States

Serviceability Limit State All structures and their foundations shall be capable of resisting the Level 1 earthquake without sustaining damage requiring post-earthquake remedial action.

Damage Control Limit State The following shall apply.

Except as required by the following clause, structures and their foundations shall be capable of resisting a Level 2 earthquake, without collapse with repairable damage, while maintaining life safety. Repairable damage to structure and/or foundation, and limited permanent deformation are expected under this level of earthquake.

Wharves and Piers on which hazardous materials are located shall be capable of resisting a Level 4 earthquake without a release of a major spill of hazardous materials.

Earthquake Load Combinations

Combination of Seismic Actions with other Load Cases Wharves and Piers shall be checked for the following seismic load combinations, applicable to both Level 1 and Level 2 earthquakes:

$$(1 + k)(D + rL) + E \qquad (1)$$

$$(1 - k) D + E \qquad (2)$$

where D = Dead Load

L = Design Live Load

r = Live Load reduction factor (depends on expected L present in actual case typically 0.2 but could be higher)

E= Level 1 or Level 2 earthquake, as appropriate.

k= 0.5 * (PGA), where PGA is the effective peak horizontal ground acceleration.

Note: seismic mass for E shall include an allowance for rL, but need not include an allowance for the mass of flexible crane structures.

Combination of Orthogonal Seismic Excitations Effects of simultaneous seismic excitation in orthogonal horizontal directions shall be considered in design and assessment of wharves and piers. For this purpose it will be sufficient to consider two characteristic cases:

$$100\%\ Ex + 30\%\ Ey \tag{3}$$

$$30\%\ Ex + 100\%\ Ey \tag{4}$$

where Ex and Ey are the earthquake (E) actions in the principal directions x and y respectively.

Where inelastic time history analyses in accordance with the requirements of Method D below are carried out, the above loading combination may be replaced by analyses under the simultaneous action of x and y direction components of ground motion. Such motions should recognize the direction-dependency of fault-normal and fault-parallel motions with respect to the structure principal axes, where appropriate.

Additional Load Combinations

Piers and wharves shall be proportioned to safely resist load combinations as shown in the following table. Each component of the structure should be analyzed for all applicable combinations. The table lists load factors to be used for each combination; the algebraic signs (+ or -) shall be those that produce the most unfavorable yet realistic loading.

$$U_i = f_D(D) + f_L(L) + f_B(B) + f_{Be}(Be) + f_C(C) + f_{Cs}(C) + f_E(E) + f_{Eq}(Eq) +$$

$$f_W(W) + f_{Ws}(Ws) + f_{RST}(R+S+T) + f_{Ice}(Ice)$$

Load Factor Design									
	U1	U2	U3	U4	U5	U6	U7	U8	U9
D Dead [1]	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
L Live [4]	1.7 [3]	1.7	1.3	1.3		1.25	[2]	1.3	
B Buoyancy	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
Be Berthing		1.7							
C Current on Structure			1.3	1.3	1.25	1.25			1.2
C_s Current on Ship			1.3	1.3	1.25	1.25			1.2
E Earth Pressure	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
Eq Earthquake							1.0		
W Wind on structure			0.3		1.25	0.3			1.2
Ws Wind on ship			0.3		1.25	0.3			
R + S + T				1.3	1.25	1.25			
Ice								1.3	1.2

R + S + T = Creep/Rib Shortening + Shrinkage + Temperature

Notes
1 A factor of 0.9 for checking members for minimal axial load and maximum moment
2 Depends on earthquake load
3 A factor of 1.3 for maximum outrigger float load from a truck crane
4 Concentrated live load

Vertical Accelerations

Except where preliminary analyses indicate special sensitivity to vertical acceleration effects such as in the case of use of batter piles, vertical accelerations need not be considered in design beyond the extent implied by use of Equations 1 and 2.

Methods Of Analysis For Seismic Response

Methods adopted for determining design forces and displacements shall be appropriate for the structural complexity of the wharf or pier under consideration, and shall include consideration of

Soil/structure interaction effects,
Natural periods of vibration of the structure,
Effects of cracking at the elastic limit state,
Reductions of stiffness resulting from inelastic action, where appropriate,
Torsional response,
Movement joints,
Gross soil deformations,
Liquefaction effects.

The primary purpose of the **analyses** will be to determine the maximum displacements expected under the design level earthquake. The primary purpose of **design** is to ensure that these displacements are compatible with the design performance limit state.

Method A: Equivalent Single Mode Analysis Where wharf structures are founded on essentially uniform foundation materials along the length of the wharf, where the ratio of wharf length to wharf width exceeds 3 and where the wharf deck may be considered to act as a rigid diaphragm, a simplified analysis involving amplification of the results from a single transverse modal response may be considered adequate for design and assessment purposes.

The design displacement for Method A is given by:

$$\Delta_D = k_a \Delta_T \qquad (5)$$

where

$$k_a = \sqrt{1 + (0.3(1 + 20e/L_L))^2} \qquad (6)$$

is an amplification factor incorporating the influence of orthogonal and torsional response effects, e is the eccentricity between the center of mass and the center of stiffness in the transverse direction, L_L is the length of the wharf segment, $_D$ is the design displacement, and $_T$ is the transverse displacement corresponding to the single mode analysis.

Method B: Multi-Mode Analysis For all structures, design displacements of the wharf or pier deck may be found by a multi-mode elastic analysis. Sufficient modes shall be considered in the analysis to capture at least 95% of the participating seismic mass in both orthogonal directions. Where multiple wharf segments of similar structure and foundation conditions are linked by shear keys, it will be conservative to consider the segments as independent “stand alone” elements, except for the estimation of shear key force levels.

Method C: Pushover Analysis For all structures, 2-D nonlinear pushover analyses shall be carried out on critical frames of wharves and piers to enable the sequence of plastic hinge formation to be determined. These pushover analyses shall be used in conjunction with the design displacements determined from Method A or Method B to establish the level of inelastic rotation developed in plastic hinges under Level 1 or Level 2 earthquakes.

Method D: Inelastic Time-History Analysis As an alternative to Methods A to C, inelastic time-history analyses may be used to determine both design displacements and inelastic rotations in plastic hinges under Level 1 and Level 2 earthquakes. A minimum of 5 spectrum compatible record sets consisting of orthogonal acceleration records shall be considered, with the mean values from the 5 analyses taken as the design or assessment levels. Each set shall have amplitude, duration and frequency appropriate for the magnitude and separation distance of the earthquake event under consideration.

Method E: Gross Foundation Deformation Analysis If geotechnical investigations indicate the possibility of gross permanent deformations of foundation material as a result of sliding on clay layers, liquefaction, or other causes, the wharf or pier shall be analyzed under the permanent foundation deformation to determine the structural displacements and internal strains and forces at critical locations.

Structural Response of Piers And Wharves

Piers and Wharves shall be designed for dependable inelastic action in accordance with the following principles:

Inelastic response of the structure shall be limited to formation of plastic rotation in carefully detailed plastic hinges in piles.

Shear failure of piles and inelastic action of deck members shall be proscribed by the implementation of capacity design principles, ensuring that the dependable strength of these members exceeds the maximum feasible input corresponding to the design flexural plastic hinging.

Joints between piles and deck members shall be designed to remain essentially elastic, with recognition of the high shear forces developed within the joint region.

Batter piles shall not be used in new design unless
(a) the piles are designed to remain elastic under Level 2 earthquake excitation, or
(b) a special study is undertaken to ensure that the structure, including the batter piles will respond within the specified performance limit state.

Note: The use of batter piles in wharves is strongly discouraged.

Structural Performance Limit State Strains

(a) Serviceability Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 1 earthquake shall not exceed:

Concrete extreme fiber compression strain:	0.004
Reinforcing steel tension strain:	0.010
Prestressing strand incremental strain	0.005

Structural Steel (pile and concrete filled pipe) 0.008

Hollow steel pipe pile 0.008

(b) Damage Control Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 2 earthquake shall not exceed:

Concrete extreme fiber compression strain:

Pile/deck hinge: Value given by equation 7, but <0.025

In-ground hinge: Value given by equation 7, but <0.008

Reinforcing steel tension strain: 0.05

Prestressing strand :
    Pile/deck hinge: 0.04

    In-ground hinge: 0.015

The design ultimate compression strain of confined concrete may be taken as

$$\varepsilon_{cu} = 0.004 + (1.4 \rho_s f_{yh} \varepsilon_{sm})/f'_{cc} \geq 0.005 \qquad (7)$$

where

- ρ_s effective volume ratio of confining steel
- f_{yh} yield stress of confining steel
- ε_{sm} Strain at peak stress of confining reinforcement, 0.15 for grade 40 and 0.12 for grade 60
- f'_{cc} Confined strength of concrete approximated by 1.5 f_c'

Structural Steel (Pile and Concrete filled pipe) 0.035

Hollow steel pipe pile 0.025

EXISTING CONSTRUCTION

The discussion of existing structures is of major importance since there are many existing terminals in use and relatively few new facilities being constructed. Many of the existing structures were built during periods when seismic standards were not well defined. In general, existing-structure performance criteria may be related to new-structure performance criteria in terms of the degree of hazard, the amount of strength required, the extent of ductility demand allowed, or the level of design ground motion. The structure once built does not "know" that it is expected to perform to a "new" or an "existing" structure criteria; it responds according the principles of structural dynamics. This guide is motivated by preservation of the environment and as such there is a mandate to use the best possible technology to ensure safe transfer of petroleum products ashore. The approach taken herein is to recognize that that the goals for both new and existing facilities should be the same. The structural parameters which are used to control the response should be the same. What is of significance is that existing structures have been in place for a period of time and have a shorter remaining life than new facilities. Thus, existing facilities face a shorter exposure to natural hazards. This major factor suggests that the design ground motions be allowed to differ based on the differing remaining life-spans of the structures. A prudent course must be charted to select reasonable goals for existing structures to minimize the adverse impact on the economics of port operations while ensuring public mandates for preservation of the environment.

The approach taken in this criteria is to utilize a factor, α, to relate the existing-structure exposure time to the new-construction exposure time taken as 50 years. The value of α is equal to or less than 1.0. The value of α is used to define the exposure time for use in the Level 1 through Level 4 earthquake return times as shown in the following sections. Determination of   establishes the seismic loading. The performance goals and response limits for existing construction remain the same as for new construction; only the loading is reduced. This applies to all elements including the structure, the foundation and all associated lifelines.

Method 1

Seismic reviews of existing waterfront construction directed by requirements of the Marine Oil Terminal Division shall utilize the above criteria for new construction as the target goal requirement for performance. In general the existing structure must satisfy the new structure performance limit states under earthquake peak ground acceleration levels corresponding to reduced exposure times as follows:

$$\text{Existing -Structure Exposure Time} = \alpha\ (\text{New-Construction Exposure Time})$$

where New-Construction Exposure Time is 50 years.

The requirement for evaluation of the seismic resistance and possible upgrade is triggered when the loading on the structure changes such as when the operation of the structure is changed or when the structure requires major repairs or modifications to meet operational needs or when recertification is required.

Determining When it is shown to be impossible or uneconomical to achieve new construction levels of performance, (α = 1), an economic/risk analysis using procedures shown in Chapter 6 of the supporting commentary shall be performed to determine the most appropriate facility exposure time and level of seismic design upgrade. Various alternative upgrade levels shall be considered ranging from the existing condition to the maximum achievable. Each alternative shall be examined to determine the cost of the upgrade, the cost of expected earthquake damage over the life of the structure and the impact of the damage on life safety, operational requirements, and damage to the environment. The choice of upgrade level shall be made by the design team based on a strategy consistent with requirements of life safety, operational needs, cost effectiveness, and protection of the environment. In no case shall be less than 0.6 of the 50-year exposure time for new construction (30 years).

As a minimum analysis shall be conducted and data developed for the cases of equal to 1.0 and 0.6. Additional cases are encouraged

Level 1- An earthquake with a 50 percent probability of exceedance in αx (50 years) exposure.

Level 2- An earthquake with a 10 percent probability of exceedance in αx (50 years) exposure.

Level 3- An earthquake with a 5 percent probability of exceedance in αx (50 years) exposure.

Level 4- An earthquake with a 3 percent probability of exceedance in αx (50 years) exposure. . **Note where ground motions from this event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows the shortest allowable return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	30	43	60
10	30	285	300
5	30	585	600
3	30	985	1000

Peer review: Review of the results of the analysis by an independent peer review panel of experts in structural engineering, geoscience, earthquake engineering, seismic risk analysis, economics, and environmental engineering/science is required. In addition, the findings from the analysis should be reviewed by the public and other stakeholders.

Method 2

Based on the recognition that an existing structure may have a fixed life and that the upgrade is intended as limited-term solution, a reduced exposure life of 25 years may be used provided facility owners by binding agreement will take the facility out of service on or before the expiration of the 25-year period. Out of service means that the structure will not be used as a marine oil terminal for transfer of hazardous materials and that all hazardous material capable of causing a spill be removed. The 25-year period begins when CSLC approves the agreement. The following levels are to be used with Method 2

Level 1- An earthquake with a 50 percent probability of exceedance in 25 years exposure. This event has a return time of 36 years and is considered a moderate event likely to occur one or more times during the life of the facility. Such an event is considered a strength event.

Level 2- An earthquake with a 10 percent probability of exceedance in 25 years exposure. This event has a return time of 237 years and is considered a major event. Such an event is considered a strength and ductility event.

Level 3- An earthquake with a 5 percent probability of exceedance in 25 years exposure. This event has a return time of 487 years and is considered a rare event. Such an event is considered a strength and ductility event.

Level 4- An earthquake with a 3 percent probability of exceedance in 25 years exposure. This event has a return time of 820 years and is considered a very rare event. Such an event is considered a containment event. . **Note where ground motions from this event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows actual return times and nominal return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	25	36	50
10	25	237	250
5	25	487	500
3	25	820	800

With mandatory removal from service before 25 year-life

Risk Analysis: In order to insure that the risk levels of a major spill are prudent, a risk analysis determining the levels of hazard and potential for a major spill shall be evaluated and submitted for review and approval.

Peer review: Review of the results of the analysis by an independent peer review panel of experts in structural engineering, geoscience, earthquake engineering, seismic risk analysis, economics, and environmental engineering/science is required. In addition, the findings from the analysis should be reviewed by the public and other stakeholders.

Allowance for Deterioration

In evaluating existing construction, it is most important to:

Evaluate the actual physical conditions of all structural members to determine the actual sizes and condition of existing members.

Provide an allowance for corrosion and deterioration.

Evaluate the properties of the construction materials considering age effects in computing yield strengths. Average actual material properties should be used in the evaluation.

Evaluate the existing structure details and connections since this is often the weakest link and source of failure.

Determine the design methodology used by the original designers at the time the structure was designed and constructed.

Evaluate displacement demands and capacities. Previous code requirements did not emphasize the need for ductility and the failure to include shear and containment reinforcing is most common in existing construction. This has led to numerous structure failures especially when batter piles have been used.

The Appendices of this report present detailed information on underwater inspection criteria and concrete repair.

GROUND FAILURE CRITERIA

Introduction

Ports and marine oil terminals are prone to a variety of geologic hazards. Of these hazards, liquefaction of the saturated, loose cohesionless soils that typically prevail at ports has been the most common source of significant damage, although other hazards – such as direct effects of ground shaking, slope instability, and tsunami – have caused extensive damage as well. Furthermore, experience demonstrates that the seismic performance of soils and port structures is strongly related to the manner in which the fills are placed and improved during construction, and also how the structures are designed and detailed to resist potential geologic hazards.

In an extensive review of the seismic performance of ports, Werner and Hung (1982) concluded that by far the most significant source of earthquake damage to waterfront structures has been pore pressure build up in loose to medium-dense saturated cohesionless soils the prevail in coastal and river environments. This observation has been supported by the occurrence of liquefaction-induced damage at numerous ports in the past decade (Werner, ed., 1998). Components of marine facilities conspicuous for poor seismic performance include: pile supported structures, sheet pile bulkheads, and gravity retaining walls founded on, or backfilled with, loose sandy soils. The generation of excess pore pressures in sandy soils can lead to phenomena associated with the loss of strength of the sandy soils (e.g., loss of bearing capacity, increase in active lateral earth pressure against retaining walls, loss of passive soil resistance below the dredgeline and/or adjacent to anchor systems, excessive settlements and lateral soil movements, buoyancy of buried tanks) contributing to the deformations of waterfront structures. In several instances, the failure of waterfront retaining structures has resulted in significant lateral ground deformations as far as 150 m into backland areas resulting in damage to buildings, tanks and buried utilities.

Sloping ground conditions exist throughout ports as natural and engineered embankments such as river levees, sand or rock dikes, etc., and as dredged channel slopes. Onshore and submarine slopes at ports have been found to be vulnerable to earthquake induced deformations. High water levels and weak foundation soils common at most ports can result in slopes that have marginal static stability and which are very susceptible to earthquake induced failures. In addition to waterfront slopes, several recent cases involving failures of steep, natural slopes along marine terraces located in backland areas have resulted in damage to port facilities. Large scale deformations of these slopes can impede shipping, damage adjacent foundations and buried structures thereby limiting port operations following earthquakes.

In addition to ground failures caused by liquefaction and weak soils marine oil terminals may be vulnerable to additional geologic hazards, as discussed in the appendix (e.g., fault movement and ground displacement, and tsunamis).

Liquefaction Hazards

Methods for evaluating the liquefaction resistance of soils are well documented and relatively simple, straightforward procedures have been adopted for use in engineering practice (Youd and Idriss, 1997). The most common methods of analyses are outlined in the commentary. These methods have been applied over the past two decades in numerous case studies and the strengths and limitations of the techniques have been well established. Although engineering evaluations for the “triggering” of liquefaction are well established, similarly well developed standard-of-practice methods for analyzing the potential consequences of soil liquefaction (i.e., extent of lateral spreading, impact on deep foundations, lifelines and structures) on waterfront components do not exist due to the complexity of these failures.

When evaluating the impact of liquefaction hazards on waterfront components the sensitivity of the structure to permanent deformations must be established. The specification of “performance goals” with respect to soil liquefaction, ground failures and possible mitigation strategies is therefore based on the allowable deformations of structures affected by the liquefaction hazards. From a practical perspective, ground deformations ranging from several inches to several feet represent failure conditions for the broad array of waterfront components at marine oil terminals. The allowable liquefaction-induced deformations of foundation soils will clearly vary with the type of component and ancillary structures, the consequence of failure, and the importance of the component on the post-earthquake operations of the terminal. The level of sophistication required for estimating the liquefaction-induced ground deformations will also vary as a function of the range of tolerable deformations, soil-structure interaction, and the configuration of the component. For example, pseudostatic, rigid body methods may be appropriate for estimating permanent deformations of earth structures affected by liquefaction, however more involved numerical procedures are recommended for liquefaction hazards involving pile supported structures. Along these lines, it should be noted that the factors of safety computed with standard stress-based liquefaction evaluation procedures and pseudo-static design methods are not adequately correlated with ground deformations to facilitate estimates of seismically-induced lateral deformations. For critical and sensitive components numerical analyses which account for the generation of excess pore pressures in foundation soils are recommended.

General Performance Objectives

Design of new structures and upgrade of existing structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas susceptible to severe ground failures if these areas cannot be economically treated with remedial soil improvement.

The presence of potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. The impact of liquefaction hazards on waterfront components shall evaluated in light of allowable deformation limits of the affected components. Since liquefaction is the primary cause of waterfront damage, remediation is a mandatory requirement where the risk of a release of hazardous materials as shown by computation is possible, such as in a pipeline break or tank failure.

Ordinary Construction - Liquefaction associated with construction categorized as “ordinary” shall be evaluated to insure the level of performance is maintained. In general ordinary construction is expected to:

Resist a moderate level of ground motion without damage;

Resist a major earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Piers and Wharves- Liquefaction assessments associated with piers and wharves shall be evaluated to insure the level of performance is maintained per the performance goals for piers and wharves stated above.

Essential Construction - Liquefaction evaluation associated with construction categorized as "essential" shall be evaluated to insure the level of performance is maintained. In general essential construction is expected to:

Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of operation/function and the structural system to remain essentially linear.

Resist the rare earthquake with a low probability of being exceeded during the life of the structure and operate/function at the level required to meet operational needs.

Hazardous Materials - Liquefaction associated with construction categorized as associated with "hazardous materials" shall be evaluated to insure the level of performance is maintained. In general construction related to containment of hazardous materials is expected to:

Resist pollution and release of a major spill of hazardous materials for a very rare event

Requisite Ground Motions For Liquefaction Evaluations

The following is based on existing guidelines (e.g., CDMG, 1997; Werner, ed., 1998), current seismic design provisions criteria and appropriate amendments to existing mandates developed for similar facilities. As previously described, the sensitivity and importance of the specific component, as well as the consequence of failure will determine the level of ground motion to be used in seismic design and analysis. The ground motions applied in liquefaction analyses will be selected or generated based on the probabilistic seismic hazard studies in accordance with the appropriate ground motion level (i.e., Level 1, Level 2, Level 3 or Level 4). The ground motions must account for the site specific dynamic response of the soils and represent the motions at depth required for the specific method of analysis.

Ordinary category of construction on average seismicity sites

For sites of average seismicity, use appropriate code provisions (e.g. NEHRP Provisions contained in FEMA, 1998).

Wharves and Piers

Design of wharves, wharf dikes, and piers shall use a two-earthquake procedure as shown above in the structural criteria section. Values less than code (NEHRP) are not to be permitted.

Essential category of construction
Sites where the structure is deemed important and essential shall use a two-earthquake procedure with a Level 1 earthquake and a Level 3 earthquake based on a local site seismicity study. Values less than code are not to be permitted.

Construction containing polluting or hazardous material
A Level 4 earthquake shall be used.

In addition to seismic ground motion there are additional hazards which must be considered:

Fault movement and local ground displacement
Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
Landslides
Tsunamis

Minimum Acceptable Methods of Analysis

Triggering of Liquefaction The following is taken verbatim from CDMG Special Publication No. 117

"If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location which might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate that those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. *Direct in situ relative density measurements, such as the Standard Penetration Test (ASTM D 1586-92) or the Cone Penetration test (ASTM D 3441-94).*
2. *Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). This is quite straightforward for waterfront sites and groundwater conditions associated with high tide levels should be used in analyses.*
3. *Non-standard penetration test data.*
4. *Geophysical measurements of shear wave velocities.*
5. *"Threshold strain" techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards are low."*

In situations where liquefaction hazards may impact marine facilities (factor of safety less than 1.5), quantitative methods of evaluating the liquefaction resistance of soils are required. The latest consensus pertaining to the evaluation of liquefaction resistance of soils has been presented by Youd and Idriss (1997). The recommended techniques for evaluation are outlined in the commentary.

Lateral Spreading The following is taken verbatim from CDMG Special Publication No. 117

"Lateral spreading on gently sloping ground is generally the most pervasive and damaging type of liquefaction failure (Bartlett and Youd, 1995). Assessment of the potential for lateral spreading and other large site displacement hazards may involve the need to determine the residual undrained strengths of potentially liquefiable soils. If required, this should be done using in-situ SPT or CPT test data (Youd and Idriss, 1997; Seed and Harder, 1990). The use of laboratory testing for this purpose is not recommended, as a number of factors (e.g., sample disturbance, sample densification during reconsolidation prior to undrained shearing, and void ratio redistribution) render laboratory testing a potentially unreliable, and, therefore unconservative basis for assessment of in-situ residual undrained strengths. Assessment of residual strengths of silty or clayey soils may, however, be based on laboratory testing of "undisturbed" samples.

Assessment of potential lateral spread hazards must consider dynamic loading as a potential "driving" force, in addition to gravitational forces. It should again be noted that relatively thin seams of liquefiable material, if continuous over large areas, may serve as significant planes of weakness for translational movements. If prevention of translation or lateral spreading is ascribed to structures providing "edge containment", then the ability of these structures (e.g., berms, dikes, sea walls) to resist failure must also be assessed. Special care should be taken in assessing the containment capabilities of structures prone to potentially "brittle" modes of failure (e.g., brittle walls which may break, tiebacks which may fail in tension). If a hazard associated with potentially large *translational movements is found to exist, then either: (a) suitable recommendations for mitigation of this hazard should be developed, or (b) the proposed "project" should be discontinued.*

When suitably sound lateral containment is demonstrated to prevent potential sliding on liquefied layers, then potentially liquefiable zones of finite thickness occurring at depth may be deemed to pose no significant risk beyond the previously defined minimum acceptable level of risk. Suitable criteria upon which to base such as assessment include those proposed by Ishihara (1985).

For information on empirical models that might be appropriate to use in these analyses, see Bartlett and Youd (1995)."

Seismically-Induced Ground Settlement The following is taken verbatim from CDMG Special Publication No. 117

"Settlements for saturated and unsaturated clean sands can be estimated using simplified empirical procedures (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1992). These procedures, developed for relatively clean sandy soils, have been found to provide reasonably reliable settlement estimates for sites not prone to significant lateral spreading.

Any prediction of liquefaction-related, or cyclically-induced, settlements is necessarily approximate, and related hazard assessment and/or development of recommendations for mitigation of such hazard should, accordingly, be performed with suitable conservatism. Similarly, it is very difficult to reliably estimate the amount of localized differential settlement likely to occur as part of the overall predicted settlement: localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made."

It should be noted that the contractive behavior of sandy soils during cyclic loading is a function of the void ratio of the soil and the in situ stresses acting on the soil. The soil will experience a reduction in volume regardless of its degree of saturation prior to ground shaking, therefore dry sands are also prone to excessive settlements during earthquake loading.

Slope Instability

The most commonly used methods for analysis of slope stability under both static and dynamic conditions are based on standard rigid body mechanics and limit equilibrium concepts that are familiar to most engineers. For use in determining the seismic stability of slopes, limit equilibrium analyses are modified slightly with the addition of a permanent lateral body force which is the product of a *seismic coefficient* and the mass of the soil bounded by the potential slip. The seismic coefficient (usually designated as k_h,

N_h) is specified as a fraction of the peak horizontal acceleration, due to the fact that the lateral inertial force is applied for only a short time interval during transient earthquake loading. Seismic coefficients are commonly specified as roughly $^1/_3$ to $^1/_2$ of the peak horizontal acceleration value (CDMG, 1997).

In most cases involving soils which do not exhibit considerable strength loss after the peak strength has been mobilized, common pseudostatic rigid body methods of evaluation will generally suffice for evaluating the stability of slopes. These methods of evaluation are well established in the technical literature (Kramer, 1996). Although these methods are useful for indicating an approximate level of seismic stability in terms of a factor of safety against failure, they suffer from several potentially important limitations. The primary disadvantages of pseudostatic methods include: (a) they do not indicate the range of slope deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils is incorporated in only a very simplified, "decoupled" manner; (c) progressive deformations that may result due to cyclic loading at stresses less than those required to reduce specific factors of safety to unity are not modeled; (d) strain softening behavior for liquefiable soils or sensitive clays is not directly accounted for; and (e) important aspects of soil-structure interaction are not evaluated.

In most applications involving waterfront slopes and embankments, it is necessary to estimate the permanent slope deformations that may occur in response to the cyclic loading. Allowable deformation limits for slopes will reflect the sensitivity of adjacent structures, foundations and other facilities to these soil movements. Enhancements to traditional pseudostatic limit equilibrium methods of embankment analysis have been developed to estimate embankment deformations for soils which do not lose appreciable strength during earthquake. Rigid body, "sliding block" analyses, which assume the that soil behaves as a rigid, perfectly plastic material, can be used to estimate limited earthquake-induced deformations. The technique, developed by Newmark (1965) is based on simple limit equilibrium stability analysis for determining the critical, or yield, acceleration which is required to bring the factor of safety against sliding for a specified block of soil to unity. The second step involves the introduction of an acceleration time history. When the ground motion acceleration exceeds the critical acceleration (a_{crit}, a_y) the block begins to move down slope. By double integrating the area of the acceleration time history that exceeds a_{crit}, the relative displacement of the block is determined. A simple spreadsheet routine can be used to perform this calculation (Jibson, 1993).

The amount of permanent displacement depends on the maximum magnitude and duration of the earthquake. The ratio of maximum acceleration to yield acceleration of 2.0 will result in block displacements of the order of a few inches for a magnitude 6 1/2 earthquake and several feet for a magnitude 8 earthquake. It should be noted that significant pore pressure increases may be induced by earthquake loading in saturated silts and sands. For these soils a potential exists for a significant strength loss. For dense saturated sand, significant undrained shear strength can still be mobilized even when residual pore pressure is high. For loose sands, the residual undrained strength which can be mobilized after high pore pressure build-up is very low and is often less than the static undrained shear strength. This may result in flow slides or large ground deformations.

Given that the sliding block analyses are based on limit equilibrium techniques, they suffer from many of the same deficiencies previously noted for pseudostatic analyses. One of the primary limitations with respect to their application for submarine slopes in weak soils is that strain softening behavior is not directly accounted incorporated in the analysis. The sliding block methods have, however, been applied for liquefiable soils by using the post-liquefaction undrained strengths for sandy soils.

In situations where the movement of a slop impacts adjacent structures, such as pile supported structures embedded in dikes, buried lifelines and other soil-structure interaction problems, it is becoming more common to rely on numerical modeling methods to estimate the range of slope deformations which may be induced by design level ground motions (Finn, 1990). The numerical models used for soil-structure interaction problems can be broadly classified based on the techniques that are used to account for the deformations of both the soil and the affected structural element. In many cases the movement of the soil is first computed, then the response of the structure to these deformations is determined. This type of analysis is termed *uncoupled*, in that the computed soil deformations are not affected by the existence of embedded structural components. A common enhancement to this type of uncoupled analysis includes the introduction of an iterative solution scheme which modifies the soil deformations based on the response of the structure so that compatible strains are computed. In a *coupled* type of numerical analysis the deformations of the soil and structural elements are solved concurrently. Two-dimensional numerical models are rather widespread in engineering practice for modeling the seismic performance of waterfront components at ports. Advanced numerical modeling techniques are recommended for soil-structure interaction applications, such as estimating permanent displacements of slopes and embankments with pile supported wharves.

Mitigation of Seismic Hazards Associated with Slope Stability

Remedial strategies for improving the stability of slopes have been well developed for both onshore and submarine slopes. Common techniques for stabilizing slopes include: (a) modifying the geometry of the slope; (b) utilization of berms; (c) soil replacement (key trenches with engineered fill); (d) soil improvement; and (e) structural techniques such as the installation of piles adjacent to the toe of the slope. Constraints imposed by existing structures and facilities, and shipping access will often dictate which of the methods, or combinations of methods, are used.

Requirements for Minimum Allowable Resistance Against Ground Failures

The design of critical structures shall include provisions for the evaluation of potential ground failures. Special care will be given to components such as tank foundations, pipe racks, and buried pipelines to preclude break resulting in release of hazardous materials. Identification of areas prone to geologic hazards is considered a necessary step in the seismic design process. Proper siting of oil terminal components is vital and it is imperative to avoid areas vulnerable to geologic hazards, or areas that cannot be economically treated with remedial ground improvement.

The presence of potentially unstable soils adjacent to oil terminal components (i.e., foundation or backfill soils) shall be fully evaluated for vertical and lateral extent, and expected seismic behavior. Specific attention shall be paid to permanent lateral and vertical ground deformations. At existing facilities, if ground failures are indicated in geotechnical evaluations of sites where the risk of a significant release of hazardous materials would result, then soil remediation, structural retrofit, or re-siting shall be considered.

The seismic performance of waterfront facilities is linked to a large degree by the magnitude of permanent ground displacements adjacent to the component. Therefore structural design provisions must be supplemented with geotechnical criteria for limiting foundation deformations during the design level earthquakes. In the following criteria liquefaction hazards are specifically addressed, however it should be understood that all forms of ground failures must be evaluated in analysis and design. It should also be emphasized that the magnitude of liquefaction induced lateral ground failures are only approximately correlated with factors of safety derived from force, or limit, equilibrium methods of analysis. In light of the fact that these rigid body methods remain the standard of practice limit, the maximum allowable ground deformations for common waterfront components are listed along with minimum factors of safety against liquefaction for foundation soils.

In the following, the allowable ground deformations are a primary design consideration and they shall be evaluated with full consideration of liquefaction hazards.

Note:

The ground deformations and factors of safety in the following sections are presented as target values. These values may be exceeded if it can be shown by reliable procedures that the performance objectives will be met. The ground deformation state must be used with the structural analysis to make certain the structural performance goals and limits are not exceeded.

It should again be noted that within each subset of components the magnitude of the ground deformations causing damage will vary. The following criteria are provided as minimum allowable conditions for insuring the acceptable seismic performance of common structures and waterfront configurations. Unique or sensitive components may require more stringent ground deformation criteria. In addition, the liquefaction criteria are considered supplementary to the deformation criteria in that the liquefaction criteria can be relaxed if it is demonstrated using appropriate methods of analysis that the deformation criteria have been met for each level of ground motion.

Ordinary Construction Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as structural collapse is avoided.

The following criteria shall be applied for ordinary construction:

Level 1 Earthquake Motions

Total settlements less than 1 inch.
Total lateral deformation of less than 3 inches.
The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

Total settlements less than 4 inches.
Total lateral deformation of less than 6 to 12 inches.
The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground deformations are within the ranges previously specified.

Wharf Dikes Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of appurtenant structures, damage to embedded deep foundations, is avoided and the structure is repairable.

The following criteria shall be applied for wharf dikes:

Level 1 Earthquake Motions

Total settlements less than 3 inches.
Total lateral deformation of less than 6 inches.
The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

Total settlements less than 6 inches.
Total lateral deformation of less than 12 inches.

The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground deformations are within the ranges previously specified.

Gravity Retaining Structures Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of the retaining structures and/or appurtenant components is avoided.

The following criteria shall be applied for gravity retaining structures:

Level 1 Earthquake Motions

Total settlements less than 3 inches at the top of the wall.

Total lateral deformation of less than 6 inches at the top of the wall.

The factor of safety against liquefaction in the foundation and backfill soils shall be greater than 1.5.

Level 2 Earthquake Motions

Total settlements less than 6 inches at the top of the wall.

Total lateral deformation of less than 12 inches at the top of the wall.

The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed wall deformations are within the ranges previously specified.

Anchored Sheetpile Retaining Walls Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of the retaining structures and/or appurtenant is avoided.

The following criteria shall be applied for anchored sheetpile retaining structures:

Level 1 Earthquake Motions

Total lateral deformation of less than 4 inches.

The factor of safety against liquefaction in the foundation and backfill soils shall be greater than 1.5.

Level 2 Earthquake Motions

Total lateral deformation of less than 10 inches.

The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed wall deformations are within the ranges previously specified.

Piers and Wharves Under Level 1 earthquake loading unacceptable deformations resulting in widespread damage to the pier and ancillary components (e.g., pipes and utility lines, pavements, conveyance equipment) should be precluded. Structural deformations may occur during a Level 2 earthquake as long as the pier or wharf, and appurtenant components, remains repairable.

The following criteria shall be applied for backfill and foundation soils adjacent to piers and wharves:

Level 1 Earthquake Motions

Total settlements less than 1 inch.

Total lateral deformation of the backfill and pier less than 3 inches.

The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

Total settlements less than 4 inches.

Total lateral deformation of the backfill and pier less than 12 inches.

The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground and structural deformations are within the ranges previously specified.

Essential Construction Under Level 1 earthquake loading deformations resulting in damage to the structure and ancillary components (e.g., pipes and utility lines, pavements, conveyance equipment) shall be precluded. Ground and structural deformations may occur during a Level 2 earthquake as long as they are limited to insure operability of critical functions in the facility. This includes utility lines associated with the structure.

The following criteria shall be applied for backfill and foundation soils adjacent to essential construction:

Level 1 Earthquake Motions

Total settlements less than 1 inch.
Total lateral deformation of the foundation and backfill soil less than 3 inches.
The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

Total ground deformations will be limited to preclude loss of operation and nonrepairable structural damage of the essential component.

Construction containing polluting or hazardous material- Settlements shall be restricted to preclude release of hazardous material causing a major spill. The computed deformation state shall be shown to have limited controlled settlements and restricted lateral spread.

SUPPORTING STRUCTURES AND LIFELINE CRITERIA

Lifelines are facility and utility systems which are vital to the operation of a terminal. They may include electric power, gas and liquid fuels, fire detection and suppression systems, telecommunications, transportation, port operation control facilities, and water supply and sewers. As stated above, safe effective seismic design consists of establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural response limits.

When considering a facility/component supporting an essential function, it is critical that the facility/component be considered as a system. It is not sufficient to consider a facility/component simply as a structure or an element, but rather it is required to consider all the elements required to accomplish the objective to be accomplished by that structure or component. This usually includes requirements for electrical power, mechanical systems, water and sewer, communications, road access etc.

Lifeline Performance Objectives

Ordinary Construction / Ordinary Lifelines -Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to

Resist a moderate level of ground motion without damage;
Resist a major level of earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Wharves and Piers Lifelines associated with pier or wharves shall be designed with the same levels of service.

Resist a moderate level of ground motion without damage;
Resist a major level of earthquake ground motion without collapse, and with the structural in a repairable condition.

Essential Construction / Essential Lifelines - Life line service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:

Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of operation/function and the structural system to remain essentially linear.
Resist the rare earthquake with a low probability of being exceeded during the life of the structure and operate/function at the level required to meet operational needs.

Note that essential lifelines can be associated with piers and wharves such as electrical control lines for valves, fire suppression, etc. In such cases the essential lifelines shall be designed to the higher essential category and provision made to account for the deformation state of the pier or wharf on the operation of the lifeline.

Hazardous Materials/Lifelines - Lifeline service associated with construction categorized as containing "hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:

Resist pollution and release of a major spill of hazardous materials for a very rare event

Provision for tanks and pipelines containing hazardous materials are discussed further below.

Design Earthquakes

The following is based on current criteria and an extension of existing mandates logically applied to analogous situations. Lifeline systems shall be designed to resist the loading produced as follows:

Ordinary category of construction on average seismicity sites
For sites of average seismicity, use code provisions contained in NEHRP which are based on an earthquake with an approximate 10 percent chance of exceedance in 50 years.

Pier or wharf category of construction

Sites where the lifeline is associated with a pier or wharf shall use a two-earthquake procedure with Level 1 and a Level 2 based on a local site seismicity study. Values less than NEHRP code are not be permitted

Essential category of construction
Sites where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 and a Level 3 based on a local site seismicity study. Values less than NEHRP code are not be permitted.

Construction containing polluting or hazardous material
A Level 4 earthquake shall be used.

Note where essential lifelines are found on piers or wharves and are required for control of hazardous materials, the highest loading shall govern.

In addition to seismic ground motion there are additional hazards which must be considered:

Fault movement and ground displacement

Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.

Landslides

Tsunamis

Modification to Design Ground Motion The ground motions used in design of lifelines may differ from the motions used in conventional facility/structure design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For lifeline component elements located within a structure, the component design loading can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component. In addition, the ground motions used for evaluations of buried lifelines should account for the depth of embedment. If a lifeline is buried at a significant depth (say 10 feet or more) then the ground surface motions should be modified to account for dynamic soil behavior.

Liquefaction And Lifelines

Design of structures shall include provisions to evaluate and resist liquefaction of foundation soils and/or backfill, and account for expected potential settlements and lateral spread deformation. Liquefaction is discussed further in following sections. Liquefaction is the single greatest cause of damage at the waterfront, especially in wharves, quaywalls and retaining structures. Special care must be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of a major spill of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas vulnerable to ground failures such as landslides and lateral spreads. The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected ground deformations (i.e., settlements and lateral earth movements) computed. Since it is rarely possible from an economic or technical perspective to eliminate earthquake induced ground deformations in waterfront environments, specific attention shall be paid to allowable ground deformations . Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformations and stresses. Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe. To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters.

Flexible couplings shall be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components.

No section of pipe shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.

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Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.

Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.

Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.

Adequate restraints shall be used for all piping.

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a pipe break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax while continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process.

Tanks

All tanks must be anchored. A pattern of well distributed anchor bolts works best compared with fewer larger bolts. A maintenance program is required to inspect the condition of the anchor bolts. Bolts showing corrosion must be replaced. Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. Tank venting is important to restrict implosion.

Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.

To achieve the required system performance and satisfy regulations, additional hazardous material containment systems are usually used as a backup. Containment systems are composed of either a singular system or a dual system as mandated by public law as discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain will need to be drained.

Design of tanks shall utilize the procedures discussed below.

Tanks shall be designed against sliding and uplift and be fully anchored.

Tanks designated as supporting essential functions (such as a fuel tank for a backup generator) shall be designed to resist Level 3 earthquakes using response spectra and the API 650 procedures.

For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment for a Level 4. This is discussed below in the section Hazardous Materials Containment.

Such spill containment requirements may be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials. This requirement will be discussed below.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Additionally stairways should not be attached to both the foundation and the tank wall.

API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API model may under predict the uplift so a value of twice the API shall be used.

Design of New Tanks The procedures described in American Petroleum Institute Standard 650 (1993 with updates through 1996) shall be used as modified and updated so as not to produce lower loads than what would be required by FEMA 302 Sec 4.1, FEMA (1998).

For essential tanks, response spectra values shall be substituted for equation values. The procedure considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period shall be substituted for ZIC_1. Additionally, sloshing period values shall be substituted for ZIC_2. Tank wall stresses are computed from overturning moments and compared with allowable values. The user shall consider the amount of tank freeboard for sloshing. Failure to provide for sloshing could damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure, and a 0.5 percent damped curve is recommended for sloshing of the liquid.

Evaluation of Existing Tanks Existing tanks shall be evaluated using the procedures for new tank design with an factor applied determine design earthquake levels. However, when an existing tank is found to be deficient it shall be checked using the procedures described by Manos (1987). Since the new tank design procedures are conservative, an existing tank may be considered as acceptable if it meets the provisions in Manos (1987) and has a lateral acceleration capacity in excess of demand.

Utilities On Piers

Piers may contain pipelines for fire suppression, freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used.

Electric Power

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All equipment shall be anchored as required. Equipment deemed as of ordinary importance shall use lateral force requirements based on provisions of the 1997 Uniform Building Code (ICBO, 1997). Equipment deemed as essential shall have the lateral force requirement computed based on local site conditions using peak ground acceleration for essential facilities (Level 3) and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.

Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and reduce or avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces. Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber. Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are often not feasible for snubber tie-down since equipment location is variable and may not be defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar; at least 4 snubbers must be used and all snubbers must be rated. Adequate accommodation of differential motion among components must be provided to prevent failure of items like ceramic insulators etc. Adequate cable slack or break away connections must be used.

Telecommunications Lifelines

Telecommunications encompasses conventional telephone requirements, communications and all equipment control lines. The equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants. Some equipment have fragility data. The equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits to prevent overturning.

Traditional damage to telecommunication equipment has included overturning of cabinet mounted electronics, failures of battery racks, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design rules must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points when the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

HAZARDOUS MATERIALS CONTAINMENT

Performance Goal

This section of the criteria is intended to address the seismic design of industrial support facilities, tanks and pipelines which contain hazardous materials. This criteria is intended to produce a level of design such that there is a high probability the facilities and components will perform at satisfactory levels and prevent a release of a major spill of hazardous material throughout their design life. Specifically for industrial support facilities, tanks and pipelines located in areas of high seismicity shall be designed:

To meet all of the provisions for tanks given above.

To resist major earthquakes, Level 4, which are considered as very rare events without release of a major spill of hazardous materials.

Design Earthquakes

The industrial support facilities, tanks and pipelines shall be designed to resist the loading produced as follows:

For sites of average seismicity, use NEHRP provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years or preferably the Level 2 event from a seismicity study if available.

Where the element/tank is deemed important and essential use a Level 3 earthquake and increase Zone Factor coefficient per response spectra techniques based on a local site seismicity study. Values less than FEMA 302 Sec 4.1 are not be permitted.

Use a Level 4 earthquake for major spill prevention.

Industrial/Hazardous Tanks and Pipelines Response At Design Loading Levels

Containment systems shall be composed of either a singular system or a dual system as mandated by public law discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems such that a failure will not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which will function should the primary system be damaged.

The structural response of the industrial support facilities, tanks and pipelines under the design earthquake levels shall meet all requirements for nonhazardous material tanks

For a Level 4 earthquake, controlled inelastic behavior with maximum ductility factors to preclude release of a major spill of hazardous materials. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

ECONOMIC / RISK ANALYSIS

Performance Objective

Marine oil terminal facilities are important facilities. Such facilities represent a huge economic investment by the company operating the facility and at the same time represent a vital resource upon which the State of California and its residents are dependent. An additional issue of pollution of the environment is of major concern. The economic viability of the operation, the need for the resources, and the concern for the environment form a basis upon which to build a framework of decision making. This criteria mandates a safe, design of new facilities and upgrade of old. It must carefully balance the three elements.

There is an increased emphasis on post-earthquake functionality of essential construction. In this light, it is important to be able to evaluate the extent and location of expected structural damage. Are there any weak links in the foundation or structural system design which will preclude operability? Operability demands that the facility be viewed as a total system not just a structural system. Utilities and the other elements must function to have operability. Additionally a procedure is required to evaluate alternative seismic designs/upgrades and select the most effective choice. This guidance presents detailed analysis procedures which can evaluate seismic strengthening, expected damage and the economics and risk of seismic design. The purpose of this procedure is to perform an economic/risk based comparison of alternative designs of a structure considering initial construction expenditures and expected earthquake induced damage over the life of the structure. It may compare different types of construction or different design levels. It is thus intended to assist the user and the design engineer in obtaining cost-effective risk-controlled seismic construction. Chapter 6 of the commentary defines the steps in the procedure for conducting an economic/risk analysis.

The extent to which an existing marine oil terminal needs to be upgraded to enhance seismic resistance depends upon the size of the risk it poses.

There are three possible approaches to seismic upgrade design:

No Consideration of Risk. Under this option, analysis of potential seismic risks would not be considered in the marine oil terminal criteria; instead, seismic risk reduction would be carried out using a conventional deterministic design or retrofit procedure that presumably meets certain seismic performance requirements under designated seismic hazard levels.

Risk as a Fallback. Position. Under this option, the user would either upgrade a facility to new construction levels of seismic performance under stringent levels of seismic hazard, or could undertake a risk analysis to justify a lower level of seismic upgrade, as long as the resulting seismic risks are acceptable.

Total Risk-Based Approach. Under this last option, all seismic risk reduction measures at a marine oil terminal would be risk based; i.e., a seismic risk analysis would be used to check whether the oil terminal system's seismic risks meet certain risk-based criteria..

It should be obvious that design of a seismic upgrade by the first option which does not consider or evaluate the risk could result in expenditures of money while the potential for a large spill may still be unacceptably high. For this reason the second option is suggested as the minimum requirement for design of an upgrade.

Oil Spill Cost and Significance

The cost of an oil spill involves several elements including: the direct cleanup cost involving the expenditures on removal of the oil, the loss of use factors, the cost of damage to the coastline and the environment in the form of the destruction of wild life and natural resources. Additionally there are third-party damages consisting of individuals who suffered property damage from contact with the oil.

The State of California Office of Oil Spill Prevention and Response estimates the cost of an oil spill based on an average of 108 oil spill incidents as follows:

Cleanup cost	\$150 /gallon
Third-part cost	\$100 /gallon
Natural resource damage	\$200 /gallon
Total Cost	\$450/gallon

Noting that there are 42 gallons per barrel, the cost of a 1200-barrel spill would be \$22,680,000. The 1990 Oil Pollution Act establishes a level of financial responsibility for a 1000-barrel oil spill in federal waters at \$35 million. It is obvious that a 1200-barrel spill is a very large and costly event. The size of a potential spill and the associated costs must be included in a risk analysis.

Outline of Risk Analysis Procedure

The risk approach is described in more detail in the Chapter 6. The major steps of the procedure, as they are given in Chapter 6, are summarized:

(1) Define system and components to be evaluated;

(2) Identify seismic risk reduction alternatives;

(3) Define multiple scenario earthquakes;

(4) Estimate site-specific seismic hazards;

(5) Implement alternative seismic design/strengthening strategies for individual components within overall system;

(6) Evaluate seismic performance of overall system; and

(7) Assess seismic risks and modify component designs if appropriate.

The specific substeps under Step 7 are summarized,

(7-1) Develop risk and decision calculations for risk reduction alternatives;

(7-2) Select risk reduction alternatives that best fit performance criteria; and

(7-3) Review selections of risk reduction alternatives with public.

The results of a economic/risk analysis are expressed in the form of cost vs. risk. The study not only shows the economics of the decision making process of selecting alternative designs, but also gives insight of component behavior showing which elements form the "weak links". The analysis quantifies the reduction in spill potential for various upgrade options. Thus the effectiveness of the economic investment for each upgrade alternative can be shown in terms of the risk of a major spill.

Supporting lifelines are part of the overall marine oil terminal system that need to be considered when evaluating whether the levels of risk to life safety, the environment, terminal operations, or economic losses are acceptable. Performance requirements given for supporting lifelines should be consistent with overall performance requirements given earlier.

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